Appendix E

Preliminary Geotechnical Evaluation

LGC
Geotechnical, Inc.

January 24, 2012 Project No. 11094-03

Mr. Larry Tucker
Baker Ranch Properties, LLC
9140 Irvine Center Drive
Irvine, CA 92618

Subject: ***Preliminary Geotechnical Evaluation of Conceptual Plan for Proposed Development of the Baker Ranch Property, City of Lake Forest, California***

In accordance with your request and authorization, LGC Geotechnical, Inc. has performed a geotechnical evaluation of the conceptual plan for proposed residential and retail development of the Baker Ranch Property, located north of the future alignment of Rancho Parkway, between Portola Parkway and Hermana Circle, within the City of Lake Forest, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed project as depicted on the conceptual plan by MVE & Partners, dated September 29, 2011.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully Submitted,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical evaluation for the proposed residential and retail use development of the Baker Ranch Property, located within the City of Lake Forest. The referenced conceptual plan (MVE & Partners, 2011) was utilized as a base plan for our Geotechnical Map (Figure 2).

The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed residential (apartments) development. As part of this report, we have: 1) reviewed readily available geotechnical reports and in-house geologic maps pertinent to the site (Appendix A); 2) performed a subsurface geotechnical evaluation of the site consisting of the excavation of six, small diameter borings with installation of one infiltration test well; 3) performed infiltration rate testing on the well and laboratory testing of soil samples obtained during our subsurface evaluation; 4) performed field mapping of the existing geologic exposures at the site; and 6) prepared this report presenting our findings, conclusions, and preliminary geotechnical recommendations for development of the proposed project.

1.2 Project Description

The project site encompasses an approximately rectangular area bounded at the south by the future alignment of Rancho Parkway, existing Portola Parkway at the east, the Foothill Transportation Corridor at the north, and industrial use areas to the west (Figure 1, Site Location Map). The current configuration of the site is a generally flat to gently sloping area that has been carved down during past uses as an earthwork borrow site (former mining operation by El Toro Materials). The site has a steep cut slope along the northern boundary that forms a small ridgeline where previously higher topography abutted the Transportation Corridor to the north. Some stockpiles, limited zones of vegetation, and a small detention basin currently exist at the southern perimeter of the central flat area of the site. The site is currently being used as staging areas for construction and landscaping companies, equipment storage, plant nurseries, and for landscape material processing.

The conceptually proposed residential and retail development is anticipated to consist of two to four story multi-family apartment buildings, a recreation center, and small area of retail use. Associated streets, flatwork, utilities, and an infiltration/detention basin at the northeast corner are also planned at the site. Approximate locations of planned uses are depicted on the conceptual site plan presented as a base for the Geotechnical Map (Figure 2).

It is our understanding that the site topography utilized as a reference for the geotechnical evaluation will require some updating prior to finalization of project grades. A grading plan review of final design grades and proposed building locations should be performed by LGC Geotechnical using topography that is current, in order to provide specific recommendations for construction of the project. Recommendations are presented herein are generalized due to lack of finished grade and future building locations.

1.3 Subsurface Geotechnical Evaluation

LGC Geotechnical has conducted a subsurface geotechnical evaluation of the site including excavation of six, small diameter hollow stem auger borings, to evaluate onsite conditions. The borings were excavated to evaluate the general engineering characteristics of the onsite materials. One of the borings was backfilled with an infiltration testing standpipe for evaluation of on-site infiltration characteristics, and the remaining borings backfilled with tamped native soils. The infiltration test well was pre-soaked overnight and infiltration testing was performed the following day.

LGC Geotechnical previously excavated two large-diameter borings within the area north of the proposed infiltration/detention basin at the northeast corner of the site during a previous phase of work at the subject site. The borings were down-hole logged by an engineering geologist for confirmation of structural geologic conditions at the site, included herein. The approximate locations of our borings are shown on the Geotechnical Map (Figure 2). Descriptions of materials encountered are provided on the boring logs, Appendix B.

1.4 Laboratory Testing

Representative bulk and driven (relatively undisturbed) samples were retained for laboratory testing during our field evaluation. Laboratory testing included in-situ moisture content and in-situ dry density (depicted on boring logs), grain size analysis, corrosion potential, direct shear, expansion index, maximum dry density and optimum moisture content (laboratory compaction) and R-Value tests.

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 99 pounds per cubic foot (pcf) to 116 pcf, with an average of 107 pcf. Field moisture contents ranged from approximately 2 percent to 14 percent, with an average of 8 percent.
- One gradation test was performed and indicated a fines content (passing No. 200 sieve) of approximately 13 percent. According to the Unified Soils Classification System (USCS) this sample is considered to be coarse-grained soil.
- Corrosion testing indicated chloride content of 51 ppm, a minimum resistivity 3650 ohm-cm, soluble sulfate content of 37 ppm and pH of 8.13.
- Expansion potential testing on two representative samples of the onsite materials indicated an expansion index of between 0 and 1, which indicates the onsite soils have “very low” expansion potential (ASTM D4829).
- Compaction testing of one bulk sample indicated a maximum dry density of 128 pcf with an optimum moisture content of 8.5 percent.
- R-Value testing on one bulk sample indicated an R-Value of 75.
- Direct shear testing was performed on one sample. The results indicate peak friction angle of 39 degrees with cohesion of 285 psf. See Section 3.1 for design shear strength parameters.

A summary of the laboratory test results are presented in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Baker Ranch Development
PROJECT NO.	11094-03
ENG. / GEOL.	DJB / KTM
SCALE	Not to Scale
DATE	January 2012

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is located within the foothills of the Santa Ana Mountains, part of the Peninsular Ranges Geomorphic Province. The region consists of dissected foothills bordering the Los Angeles Basin to the northwest and the granite-core Santa Ana Mountains to the east. The Southern California Batholith forms the core of the Santa Ana Mountains, which is overlain by a thick sequence of sedimentary units, which comprise the foothills. The foothills have been tilted, folded, and faulted since deposition as a result of regional uplift. Drainage from the nearby mountains has dissected the subject area and the alluvial deposits within the subject region are ultimately connected to alluvium of the Tustin Plain to the southwest of the site. Late Miocene to Early Pliocene bedrock materials of the Capistrano Formation that underlie the subject site are primarily composed of sandstone and silty sandstone.

2.2 Site-Specific Geology

The bedrock geologic unit mapped on the site is the Oso Member of the Tertiary-aged Capistrano Formation. The material is best exposed in the relatively long and steep cut slope that bounds the northern boundary of the site. A zone of Quaternary young fan deposits that reportedly previously covered the majority of the site (Morton, 2004) remains at the west portion of the site as encountered in boring HS-6; the majority of that unit was likely removed with the earthwork borrow site activity that has occurred at the site over the past years. Additionally, limited zones of surficial materials including undocumented fill, stockpiles of materials and tailings, and zones of colluvium are anticipated to be encountered at isolated locations throughout the subject site. A brief description of the materials encountered during drilling and surface mapping is presented in the following section, and the approximate lateral extents are depicted on the site Geotechnical Map (Figure 2).

Based on our review of the State of California Seismic Hazard Zones El Toro 7.5 Minute Quadrangle (CDMG, 2001), a limited zone depicting potential for earthquake-induced landslide is within the eastern limits of the subject site; however that determination was based on the topography that existed prior to the current topography, that has since been generally flattened by comparison with the extensive soil export that has occurred at the site over the past years. Zones of potential liquefaction are not depicted within the limits of the proposed grading. No indication of onsite faulting was observed during our evaluation.

2.2.1 Capistrano Formation – Oso Member (Map Symbol - Tco)

The Oso Member of the Tertiary Capistrano Formation is exposed across much of the site and underlies the entire site at depth. The Oso Member was deposited in a submarine fan complex environment. As encountered, these materials generally consist of fine to coarse, slightly moist, very dense, sandstone and silty sandstone. The material is generally light gray to off white in color.

2.2.2 Quaternary Young Fan Deposits (Map Symbol – Qyf)

The Quaternary Young Fan Formation is exposed at the west portion of the site where a relatively limited amount of the material has been left in place over the bedrock formation that underlies the entire site. The young fan deposits likely covered a larger area of the site prior to initiation of the mining operation. As encountered, these materials generally consist of variable, mottled light brown and light orange, moist, medium dense, silty fine sand.

2.3 Geologic Structure

The site bedrock consists of a series of layered sedimentary lithologies that have been tilted through regional tectonics. In general, the Oso Member material was found to be moderately bedded, consistently dipping approximately 13 to 21 degrees to the west with minor cross bedding dipping to the east from 14 to 22 degrees, as depicted on the Geotechnical Map (Figure 2). No regional foliation and/or fracturing and jointing trend have been observed on the site. No indication of onsite faulting was observed during our evaluation.

2.4 Ground Water

During our subsurface evaluation, no groundwater was encountered to an elevation of 747 feet above mean sea level. During site grading, it should be expected that ground water may be locally encountered in perched conditions within the bedrock during grading near the proposed new detention basin to be located in the northeast corner of the site (LGC Geotechnical, 2012). However, ground water is not anticipated to be a major constraint for the proposed grading or site development.

Seasonal fluctuations of ground-water elevations should be expected over time. In general, ground-water levels fluctuate with the seasons and local zones of perched ground water may be present within the near-surface deposits due to local seepage or during rainy seasons. Local perched ground water conditions or surface seepage may develop once site development is completed and landscape irrigation commences.

2.5 Infiltration Testing

It is our understanding that the site will be provided with a detention/infiltration basin in general accordance with the referenced plan by Psomas (2011), as evaluated in the referenced LGC Geotechnical report (2012). Infiltration testing was performed at one location in the northeastern portion of the site; see Geotechnical Map, Figure 2. During our subsurface evaluation of the subject site, Boring HS-4 was provided with an infiltration testing well, and testing was performed after pre-soaking the installation the previous day.

Intentional infiltration of surface water is anticipated to magnify perched ground water conditions. Additional recommendations such as vertical drywells for control of subsurface water should be considered once site grading plans are finalized, as discussed in Section 5.7.

2.6 Faulting

California is located on the boundary between the Pacific and North American Lithospheric Plates. The average motion along this boundary is on the order of 50-mm/yr in a right-lateral sense. The majority of the motion is expressed at the surface along the northwest trending San Andreas Fault Zone with lesser amounts of motion accommodated by sub-parallel faults located predominantly west of the San Andreas including the Elsinore, Newport-Inglewood, Rose Canyon, and Coronado Bank Faults. Within Southern California, a large bend in the San Andreas Fault north of the San Gabriel Mountains has resulted in a transfer of a portion of the right-lateral motion between the plates into left-lateral displacement and vertical uplift. Compression south and west of the bend has resulted in folding, left-lateral reverse thrust faulting, and regional uplift creating the east-west trending Transverse Ranges and several east-west trending faults. Further south within the Los Angeles Basin, “blind thrust” faults are believed to have developed below the surface also as a result of this compression, which have resulted in earthquakes such as the 1994 Northridge event along faults with little to no surface expression.

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults. The result is the Alquist-Priolo Earthquake Fault Zoning Act, which was most recently revised in 1997 (Hart, 1997). According to the State Geologist, an active fault is defined as one, which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). A potentially active fault is defined as any fault, which has had surface displacement during Quaternary time (last 1,600,000 years), but not within the Holocene. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from the zone of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. The site is located approximately 16 kilometers (10 miles) from the Elsinore Fault. The possibility of damage due to ground rupture is considered low since active faults are not known to transect the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching and shallow ground rupture, soil liquefaction, dynamic settlement, seiches, and tsunamis. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. The closest major active faults that could produce these secondary effects include the Newport Inglewood Offshore, Elsinore and Whittier Faults. A discussion of these secondary effects is provided in the following sections.

The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

2.6.1 Lurching and Shallow Ground Rupture

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are not likely to be significant where the thickness of soft sediments does not vary appreciably under structures. Minor cracking of near-surface soils, due to shaking from distant seismic events, is not considered a significant hazard, although it is a possibility at any site.

2.6.2 Liquefaction and Dynamic Settlement

Liquefaction and liquefaction-induced dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Liquefaction is typified by a build-up of pore-water pressure in the affected soil layer to a point where a total loss of shear strength occurs, causing the soil to behave as a liquid. Liquefaction primarily occurs in loose, saturated, granular soils while cohesive soils such as silty clays and clays are generally not considered susceptible to soil liquefaction. The effect of liquefaction may be manifested at the ground surface by rapid settlement and/or sand boils. Based on our review of the State of California Seismic Hazard Zones El Toro 7.5 Minute Quadrangle (CDMG, 2001), no zones having a potential for liquefaction have been depicted within the proposed limits of grading. Based on the proposed finish grades, depth of compacted fill, and lack of a shallow ground-water table, the potential for post construction liquefaction and liquefaction-induced settlement is considered very low.

2.6.3 Lateral Spreading

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the low potential for liquefaction, the potential for lateral spreading is also considered very low.

2.7 Rippability

Based on the excavation characteristics encountered during our subsurface evaluation, rippability of the surface bedrock is not anticipated to be an issue during site grading and construction. It is anticipated that the onsite materials may be excavated with conventional heavy-duty construction equipment in good working condition.

3.0 ANALYSIS

3.1 Slope Stability Analyses

The soil shear strength parameters provided below are based upon our laboratory testing of the onsite materials and published shear strength data (CDMG, 2000b). Laboratory test results are provided in Appendix C. The raw data obtained from the direct shear test performed as part of this study and previous studies at the site are provided in Appendix C, with respect to the design values for Capistrano Formation – Oso Member bedrock. The design static shear strength values shown below were selected based on existing and previous laboratory test results and engineering judgment.

TABLE 1

Static Soil Shear Strength Parameters for Slope Stability Analysis

	ϕ (Degrees)	**Cohesion (psf)**
Capistrano Formation – Oso Member Bedrock (Tso)	32	100
Compacted Fill	32	50

3.2 Surficial Stability

There is some risk of surficial instability during periods of heavy rainfall for newly constructed cut and fill slopes which contain either Capistrano Formation - Oso Member bedrock or fill derived from the Oso Member, as these materials have a very low cohesion value (50 to 100 psf). Our experience with slopes made up of similar sandy materials with low cohesion indicates that during periods of heavy rain within 1 to 2 years after construction and prior to the establishment of deep rooted vegetation cover, extensive erosion may occur. Therefore, we recommend that any cut or fill slopes be immediately planted and irrigated once constructed, as vegetation has a positive effect on surficial stability. See Section 5.2 for discussion on treatment of cut slopes to increase vegetation growth. Additionally, we recommend that grading be completed after the rainy season to further reduce the potential for surficial instabilities. If the slopes are constructed prior to the rainy season, additional recommendations include the use of the jute netting or a spray-on type of application to reduce the potential for surficial instabilities.

If the risk of surficial instability is not acceptable to the owner, the site can be selectively graded and cohesive material placed on the outer 15 feet of all slopes to increase the surficial stability. Based on our review of the site, this is anticipated to be extremely costly, if not impossible, due to the lack of onsite cohesive material.

3.3 *Seismic Design Criteria*

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2007 C.B.C. Site coordinates of latitude 33.6673 degrees north and longitude -117.6542 degrees west, which are representative of the site, were utilized in our analyses. The initial results of our analyses for the maximum considered earthquake (MCE) spectral response accelerations (S_S and S_1) are presented on the Table 2A.

TABLE 2A

Seismic Design Parameters

Selected Parameters from the 2007 C.B.C. Section 1613 - Earthquake Loads	**Seismic Design Values**
Site Class per Table 1613.5.2	C
Spectral Acceleration for Short Periods (S_S)*	1.383 g
Spectral Accelerations for 1-Second Periods (S_1)*	0.496 g
Site Coefficient F_a per Table 1613.5.3(1)	1.0
Site Coefficient F_v per Table 1613.5.3(2)	1.304

* Calculated from the USGS computer program “Seismic Hazard Curves, Response Parameters and Design Parameters” v5.0.9a (10/21/09)

The spectral response accelerations (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}), adjusted for Site Class C, were evaluated for the site in general accordance with section 1613 of the 2007 C.B.C. These site class adjusted parameters are listed on Table2B.

TABLE 2B

<u>Seismic Design Parameters Modified for Site Class C</u>

Selected Parameters from the 2007 C.B.C. Section 1613 - Earthquake Loads	**Seismic Design Values Modified for Site Class C**
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class C [Note: $S_{MS} = F_a S_S$]	1.383 g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class C [Note: $S_{M1} = F_v S_1$]	0.647 g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class C [Note: $S_{DS} = (^2/_3) S_{MS}$]	0.922 g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class C [Note: $S_{D1} = (^2/_3) S_{M1}$]	0.431 g

In accordance with Table 1613.5.6 (1, 2), the seismic design category for the subject site is Category D, where $S_{DS} \geq 0.5$ and $S_{D1} \geq 0.2$.

Section 1802.2.7 of the 2007 C.B.C. states that the PGA for a site may be defined as $S_{DS}/2.5$. The S_{DS} for the subject site has been calculated as 0.922 g.

Therefore, PGA = 0.922/2.5 = **0.37 g.**

4.0 CONCLUSIONS

Based on the results of our subsurface geotechnical evaluation and geotechnical review of the proposed drainage plans, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations contained in the following sections are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- The bedrock geologic unit mapped on the site is the Oso Member of the Tertiary-aged Capistrano Formation. Surficial units consisting of Quaternary Young Fan Deposits and scattered areas of stockpiled materials and colluvium overlie the bedrock material.
- Ground water was not encountered in the subsurface investigation. Ground water is not anticipated to be a major constraint to the proposed grading and development. However, isolated areas of perched ground water should be anticipated during grading and/or after construction due to project intent to infiltrate low-flow surface water to the subsurface.
- Active or potentially active faults are not known to exist on or in the immediate vicinity of the site.
- The main seismic hazard that may affect the site is from ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life. The estimated peak horizontal ground acceleration is 0.37 g.
- Based on the proposed finish grades, depth of compacted fill, and lack of a shallow ground-water table, the potential for post-construction liquefaction and liquefaction-induced settlement is considered very low.
- Based on the results of our evaluation, it is anticipated that the onsite bedrock materials may be excavated with conventional heavy-duty construction equipment in good working condition.
- Proposed cut and fill slopes are generally anticipated to be geotechnically acceptable as long as they are constructed in accordance with project recommendations and our General Earthwork and Grading Specifications (Appendix D); however there is potential for surficial instability, or erosion, due to low cohesion of on-site materials. Final design of interior and perimeter slopes should be geotechnically reviewed at the grading plan review phase of work.
- Because there is a long term and short term risk for surficial slope instability (erosion rills and shallow slumping) associated with cut and fill slopes constructed with materials available on-site, we recommend that the completed cut and fill slopes be immediately planted and irrigated, as vegetation has a positive effect on surficial stability.
- Based on the results of limited laboratory testing, site soils are anticipated to have a very low expansion potential. This should be confirmed at the completion of grading.
- Based on the results of limited laboratory testing, site soils have a negligible sulfate exposure condition to concrete in direct contact with the onsite soils. This should be confirmed at the completion of grading.
- From a geotechnical perspective, the existing onsite soils are suitable material for use as fill, provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.

5.0 PRELIMINARY RECOMMENDATIONS

The following recommendations are to be considered preliminary, and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the City of Lake Forest.

It should be noted that the following geotechnical recommendations are intended to provide the City of Lake Forest with sufficient information to develop the site in general accordance with the 2007 C.B.C. requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an “acceptable level”. The “acceptable level” of risk is defined by the California Code of Regulations as “that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project” [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvements may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, ground-water seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

All geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

5.1 Site Earthwork

We anticipate that earthwork at the site will consist of removal of existing improvements associated with the former and current land use (including off-site disposal of any organic-related materials, i.e. mulch, etc.), clearing and grubbing, rough grading, precise grading and construction of the proposed new improvements, including the residential structures, recreation center, subsurface utilities, interior streets, parking areas, etc. Rough grading is anticipated to include design cuts and fills, removal of potentially compressible materials (where present), overexcavation of the cut portion of transitional pads, construction of a fill slope along the currently existing oversteep slope at the northern perimeter of the site, and construction of interior slopes, among other tasks.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the City of Lake Forest grading requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix D. In case of conflict, the following recommendations shall supersede those included in Appendix D. The following recommendations should be considered preliminary and may be revised within the future grading plan review report, or based on the actual conditions encountered during site grading.

5.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of surface obstructions and potentially compressible material (such as stockpiled materials, young fan deposits, colluvium, and vegetation). Vegetation and debris should be removed and properly disposed of offsite. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material.

5.1.2 Removal and Recompaction

All potentially compressible materials not removed by the planned design cuts should be excavated to competent material and replaced with compacted fill soils. Given that the majority of the site has Oso formation at (or very near) existing grades, the amount of remedial grading is expected to be small. For budgetary purposes, we anticipate remedial grading on the order of 5 feet in the mapped young fan deposits and colluvium areas shown on the Geotechnical Map, Figure 2. It is our understanding that the existing stockpiled materials will be removed from the site prior to the start of grading. In areas where existing tenants are present (approximately 50 percent of the site), up to a 1-foot removal is recommended in order to remove any disturbed/remnant surficial materials. The remainder of the site, which is a "true cut" area, we recommend the exposed materials be ripped 6 inches, moisture conditioned, and recompacted in place. The actual depth of remedial grading will be determined in the field based on the exposed conditions. Depending on the final site grades and building locations, additional grading is anticipated in order to remove any potential cut/fill transition in accordance with Section 5.1.5.

From a geotechnical perspective, material that is removed (young fan deposits, stockpiles, colluvium, etc.) may be placed as fill provided that the material is relatively free of organic material and/or deleterious debris, is moisture-conditioned or dried (as needed) to obtain near-optimum moisture content, and then recompacted. Removal bottoms should be observed and accepted by LGC Geotechnical prior to fill placement. Areas to receive fill and/or other surface improvements should be scarified, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Society for Testing and Materials [ASTM] Test Method D1557).

5.1.3 Subdrains

A subdrain should be constructed at the heel of potential fill slopes at the northern boundary of the site in order to control groundwater. Specific recommended locations for the fill slope subdrain and any other recommended subdrain should be provided with a grading plan review of finalized project plans. Subdrains should be constructed in accordance with the recommendations provided in Appendix D**.**

A representative of the project civil engineer should survey the installed subdrains for alignment and grade prior to fill placement above the subdrains.

5.1.4 Fill Placement

From a geotechnical perspective, the onsite soils are generally suitable for use as compacted fill, provided they are screened of significant organic materials and construction debris. Areas prepared to receive structural fill and/or other surface improvements should be scarified, brought to at least optimum-moisture content, and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). Material to be placed as fill should be brought to above optimum moisture content and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, granular fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material (material larger than 8 inches in maximum dimension) should be placed in accordance with the recommendations provided in Appendix D.

From a geotechnical viewpoint, import soils (if necessary) should consist of clean, granular soils of very low expansion potential (expansion index 20 or less based on U.B.C. 18-2). Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of 48 hours prior to any planned importation.

5.1.5 Overexcavation of Cut/Fill Transitions

To reduce the potential for differential settlement below the proposed buildings or other sensitive improvements, we recommend the cut portion of cut/fill transitions be overexcavated to provide a minimum of 2 feet of fill below the proposed footings, and extending at least 3 horizontal feet beyond. The overexcavated material should then be replaced by compacted fill material to design grade.

5.1.6 Trench Backfill and Compaction

The onsite materials may generally be considered suitable for use as trench backfill, provided that they are screened of rocks and other material greater than 6 inches in diameter and significant organic matter. If trenches are shallow or the use of conventional equipment may result in damage to the utilities, a clean sand having a SE > 30 may be used to bed and shade the pipes. Sand backfill may be densified by jetting or flooding and then tamping to ensure adequate compaction. Otherwise, trench backfill should be compacted in thin uniform lifts by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557). A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project specifications.

5.1.7 Shrinkage and Bulking

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. Table 3 is an estimate of shrinkage and bulking factors for the geologic units found on the site. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction achieved during grading.

It should be stressed that these values are only estimates and that an actual shrinkage factor is extremely difficult to predetermine. The effective shrinkage of onsite materials will depend primarily on the type of compaction equipment and method of compaction used by the contractor.

TABLE 3

Shrinkage and Bulking Factor

Geologic Unit	Shrinkage	Bulking
Tco – below 2 feet	-	5 to 10 %
Stockpile (if present)	15 to 20 %	-
Qyf	5 to 10 %	-

The above shrinkage, bulking, and subsidence estimates are intended as an aid for the project civil engineer in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual shrinkage/bulking that occurs during grading. To avoid potential import/export during rough grading, we recommend the civil engineer incorporate a balance area (i.e. recreation area or other area) which can accommodate the actual amount of soil that is present.

5.2 Slope Stability

5.2.1 Cut Slopes

The northern boundary of the subject site currently has an oversteep cut slope ranging between 15 and 40 feet high. We anticipate the western approximately two-thirds of the slope will be provided with a stabilization fill keyway in order to create a 2:1 (H:V) or shallower inclination. The remaining third of the slope may or may not be designed as a cut slope after considerations by the owner and geotechnical review of final project design plans.

Since the presence of vegetation will increase the long term surficial factor of safety, from a plant growth perspective, it may be desirable to overexcavate the face of such a cut slope and replace it with compacted fill. The idea being that since the vegetation would be planted in fill instead of bedrock, the growth rate of the plants will be better. However, from a geotechnical perspective, in the short term the factor of safety against surficial failure decreases as the fill is slightly weaker than the bedrock. The decision regarding whether or not to overexcavate the cut slopes should be made by the owner based on information provided by the project landscape architect.

If the cuts slopes are to be overexcavated and replaced with fill, they should be constructed as replacement fill slopes in accordance with the recommendations provided on our Stabilization Fill detail provided in Appendix D. Properly outletted back drains should be constructed along stabilization fill backcuts.

In general, to reduce the potential for backcut failures, we recommend the keyway backcuts be planned to minimize the time the backcut is left exposed. The backcuts should not be initiated prior to forecasted rain or where they will be left open for extended periods.

Backcuts and key excavations should be geologically mapped by the geotechnical consultant during excavation to confirm the anticipated conditions. If adverse joints, fractures, and/or bedding are exposed, additional analysis and/or remediation measure may be required. The grading contractor must trim the backcuts with a slope board to remove loose material to allow for confirmational mapping.

5.2.2 Fill Slopes

Fill slope faces should also be compacted to minimum project specifications. This may require overbuilding of the slope face and trimming back to design grades. To improve surficial stability, vegetation specified by the landscape architect should be established on the slope face as soon as it is practical.

5.3 Conventional Foundation Recommendations

Based on the site geotechnical conditions and assuming the remedial recommendations provided herein are appropriately implemented, the site may be considered suitable for the support of the proposed structures using a conventional foundation system. As the expansion index is less than 20 (very low). Minimum footing depths should be 18 inches for two-story buildings. Slab subgrade should be kept at optimum moisture content to a minimum depth of 12 inches. Structural steel reinforcement should be designed by the structural engineer based on the geotechnical parameters contained herein. See Section 5.4 for bearing values.

5.4 Soil Bearing

An allowable soil bearing pressure of 2,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 18 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment of 100 psf for each additional foot of foundation width to a maximum value of 2,500 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

In utilizing the above-mentioned allowable bearing capacity, foundation settlement due to structural loads is anticipated to be less than ½-inch over a horizontal span of 40 feet.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.30 may be assumed with dead-load forces. A passive lateral earth pressure of 300 psf per foot of depth (or pcf) may be used for the sides of footings poured against properly compacted fill. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

Bearing values indicated above are for total dead loads and frequently applied live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces. The passive pressure may be increased by one-third due to wind or seismic forces.

5.5 <u>Lateral Earth Pressures for Preliminary Retaining Wall Design</u>

The following parameters are applicable for conventional retaining walls that will be less than 6 feet in height.

Lateral earth pressures for select material or approved native soils, meeting indicated project specifications, are provided below. Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft of depth or pcf. These values do not contain an appreciable factor of safety, so the civil and/or structural engineer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 125 pcf may be assumed for calculating the actual weight of soil over the wall footing. The retaining wall designer should clearly indicate on the retaining wall plans the type of backfill (select or native) used in the retaining wall design.

The following lateral pressures for approved free-draining granular soils (sand equivalent (SE) of 30 or greater per CTM 217 and Expansion Index (EI) not greater than 20 per test method U.B.C. 18-2) for level or sloping backfill are presented on Table 4.

TABLE 4

Lateral Earth Pressures – Approved Select Material

Conditions	**Equivalent Fluid Unit Weight (pcf)**	
	Level Backfill	**Seismic Earth Pressures (pcf)***
	Approved Soils	**Approved Soils**
Active	35	17
At-Rest	60	5
Passive	300	-

* Per Section 1803.5.12 of the 2010 CBC, the seismic earth pressures given below are applicable to "structures assigned to Seismic Design Category D, E, or F in accordance with Section 1613." All static and seismic loads shall be taken as a "normal", triangular distribution with the resultant acting at H/3 in relation to the base of the retaining wall footing (where H is the retained height). The incremental seismic loads were determined in general accordance with the standard of practice in the industry using the Mononobe-Okabe (1929) method and confirmed by Dr. Nickolas Sitar from UC Berkeley (personal communication, 2011).

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at-rest" conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance. The passive earth pressure values assume sufficient slope setback criteria.

The equivalent fluid pressure values assume free-draining conditions. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer. Surcharge loading effects from the adjacent structures should be evaluated by the geotechnical and structural engineer. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence and is not a substitute for waterproofing. Efflorescence is generally a white crystalline powder (discoloration) that results when water, which contains soluble salts, migrates over a period of time through the face of a retaining wall and evaporates. If seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential. The outlet pipe should be sloped to drain to a suitable outlet. Typical wall drainage design is illustrated in Figure 3 (Typical Retaining Wall Backfill and Drainage Detail, Rear of Text).

For sliding resistance, the friction coefficient of 0.30 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads.

Excavations should be made in accordance with Cal/OSHA, as a general guideline. The backfill soils should be compacted to at least 90 percent relative compaction (based on ASTM Test Methods D2922 and D3017). Prolonged exposure of back-cut slopes during construction may result in some localized slope instability. Excavation safety is the sole responsibility of the contractor.

Soil bearing values for shallow footings are provided in Section 5.4.

5.6 ***Control of Surface Water and Drainage Control***

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 2 percent for earthen surfaces for a distance of at least 5 feet, and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets.

5.7 ***Infiltration Recommendations***

Recent regulatory changes have occurred that promote that storm water be infiltrated to the subsurface rather than directed to conventional storm drainage systems. These infiltration systems are variable, but typically include permeable pavement/pavers, grass lined swales, detention basins and/or drywells. It should be noted that intentionally infiltrating storm water to the subsurface directly conflicts with the geotechnical engineering objective of ensuring satisfactory performance of foundations, by preventing surface water infiltration. The geotechnical stability and integrity of the project site is reliant upon appropriately handling all surface water.

In general, the vast majority of geotechnical related problems that occur are directly related to improper drainage. Distress in the form of movement of foundations and other improvements could occur as a result of saturation and loss of soil support of foundations and pavements, settlement, collapse, internal soil erosion, and/or expansion. Additionally, off-site properties and improvements may be subjected to seeps, springs, slope instability, movements of foundations or other impacts as a result of water infiltration. In addition, water may enter underground utility pipe zones and impact other improvements which are located at a lower elevation.

Although we do not recommend infiltration of surface water, the subject site is planned to have a detention basin at the northeast portion of the site as depicted on the conceptual plan utilized as a base for the Geotechnical Map (Figure 2). It is our understanding that the detention basin may be constructed prior to construction of the residential site proposed herein (Psomas, 2012 & LGC, 2012). We recommend a design infiltration rate of 3 inches/hour. This design rate includes a factor of safety of 2.0. Due to the proximity of the proposed improvements to the detention basin, consideration should be given to installing drywells. These will help promote deeper infiltration of surface water and will reduce the potential for nuisance type water issues.

As with all systems designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be anticipated regardless.

Any future changes that warrant a geotechnical review should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been appropriately incorporated.

5.8 Preliminary Pavement Recommendations

Laboratory testing of samples of the onsite materials collected during our filed work indicate an R-value of 75. Based on a design R-value of 50, we recommend the following provisional minimum street sections for Traffic Indices of 4.5 through 6.0. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final street sections should be confirmed by the project civil engineer based upon the projected Traffic Index.

TABLE 5

Paving Section Options

Assumed Traffic Index	4.5 through 5.5	6
R-Value Subgrade	50	50
AC Thickness	4.0 inches	4.0 inches
Base Thickness	4.0 inches	5.0 inches

The thicknesses shown are for minimum thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Aggregate base should conform to the requirements of the latest edition of the Standard Specifications for Public Works Construction (“Greenbook”). Aggregate base should be compacted to a minimum of 95 percent relative compaction over subgrade compacted to a minimum of 90 percent relative compaction per ASTM- D1557.

5.9 Corrosivity to Concrete and Metal

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants as they determine necessary. It should be noted that chloride test results indicate approximately 51 parts per million (ppm), sulfate content is approximately 37 ppm (less than 0.10 percent) and pH is approximately 8.1. Caltrans defines a corrosive area as where any of the following exist: 1) the soil contains more than 500 ppm of chloride; 2) more than 2,000 (0.2 percent) of sulfate; or 3) a pH less than 5.5. Therefore, preliminary test results indicate the onsite soils are non-corrosive.

Based on preliminary sulfate testing performed at the site, concrete should be designed in accordance with the negligible category (2007 C.B.C). These findings should be confirmed at the end of grading.

5.10 Nonstructural Concrete Flatwork

Concrete flatwork (such as sidewalks, patios, etc.) has the potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 7. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting.

TABLE 6

Nonstructural Concrete Flatwork for Low Expansion Potential

	Sidewalks	**City Sidewalk Curb and Gutters**
Minimum Thickness (in.)	4 (nominal)	Per City of Lake Forest
Presaturation	Wet down prior to placing	Per City of Lake Forest
Reinforcement	2 No. 3 Rebar longitudinal	Per City of Lake Forest
Thickened Edge (in.)	—	Per City of Lake Forest
Crack Control Joints	Saw cut or deep open tool joint to a minimum of $^{1}/_{3}$ the concrete thickness	Per City of Lake Forest
Maximum Joint Spacing	8 feet	Per City of Lake Forest
Aggregate Base Thickness (in.)	—	Per City of Lake Forest

5.11 Construction Observation and Testing

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. Once the grading plans have been finalized and the mining operations in the northern portion have ceased, LGC Geotechnical should conduct a site visit and perform a grading plan review. Updated recommendations and/or additional field work (may be necessary).

The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Construction observation and testing should also be performed by LGC Geotechnical during future grading, excavations, backfill of utility trenches, preparation of pavement subgrade and placement of aggregate base, foundation or retaining wall construction, or when any unusual soil conditions are encountered at the site. Foundation plans and final project drawings should be reviewed by this office prior to construction.

6.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification.